GROUND ENGINEERING

# Geotechnical Evaluation
# Dental Office Building—Lot 4
# Berthoud, Colorado


Prepared For:

## KJW Real Estate LLC

**329 Bronco Court**
**Berthoud, Colorado 80513**

**Attention: Ms. Justine Wolfenden**

**Job Number: 24-3033** **August 29, 2024**

## TABLE OF CONTENTS

## PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) in support of design and construction of the proposed dental office building planned for construction within Lot 4 of the Amended Gateway Park First Filing – Phase 2 – Replat D, southwest of Lake Avenue and Berthoud Parkway in Berthoud, Colorado. Our study was conducted in general accordance with GROUND's Proposal Number 2407-1388 dated July 16, 2024, between KJW Real Estate, LLC and GROUND.

A field exploration program was conducted to obtain information on the subsurface conditions. Material samples obtained during the subsurface exploration were tested in the laboratory to provide data on the classification and engineering characteristics of the on-site soils. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained and to present our findings and conclusions based on the proposed development/improvements and the subsurface conditions encountered. Design parameters and a discussion of engineering considerations related to the proposed improvements are included herein. This report should be understood and utilized in its entirety; specific sections of the text, drawings, graphs, tables, and other information contained within this report are intended to be understood in the context of the entire report. This includes the *Closure* section of the report which outlines important limitations on the information contained herein.

This report was prepared for design purposes of KJW Real Estate, LLC, based on our understanding of the project at the time of preparation of this report. The data, conclusions, opinions, and geotechnical parameters provided herein should not be construed to be sufficient for other purposes, including the use by contractors, or any other parties for any reason not specifically related to the design of the project. Furthermore, the information provided in this report was based on the exploration and testing methods described below. Deviations between what was reported herein and the actual surface and/or subsurface conditions may exist, and in some cases those deviations may be significant.

## PROPOSED CONSTRUCTION

Based on provided information[1] and the provided survey,[2] we understand a single-story, wood-framed building (approximately 4,500 square feet in footprint area) is planned for construction. We assume no below grade levels are planned. New buried utilities and local landscaping are anticipated as part of the new facility.

We understand no new pavements are planned as part of the proposed construction; therefore, no pavements were addressed as part of this report.

**If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed above, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to reevaluate the conclusions and parameters presented herein.**

***Performance Expectations*** Based on our experience with other, similar projects, we assume that post-construction, building foundation and floor movements on the order of 1 inch are acceptable to, and anticipated by KJW Real Estate, LLC, as are the resultant distress and maintenance measures. Similarly, we anticipate that movements of somewhat greater magnitude (1 to 2 inches) are acceptable and anticipated for flatwork, although movement estimates closer to 1 inch may be preferable near the buildings. GROUND will be available to discuss the risks and remedial approaches outlined in this report, as well as other potential approaches, upon request if post-construction movements of these magnitudes are not acceptable and anticipated.

---

[1] Tait and Associates (2023) *Amended Gateway Park First Filing—Phase 2—Replat D. A portion of the southeast ¼ of section 15, township 4 north, range 69 west of the 6th P.M., Town of Berthoud, County of Larimer, State of Colorado.*

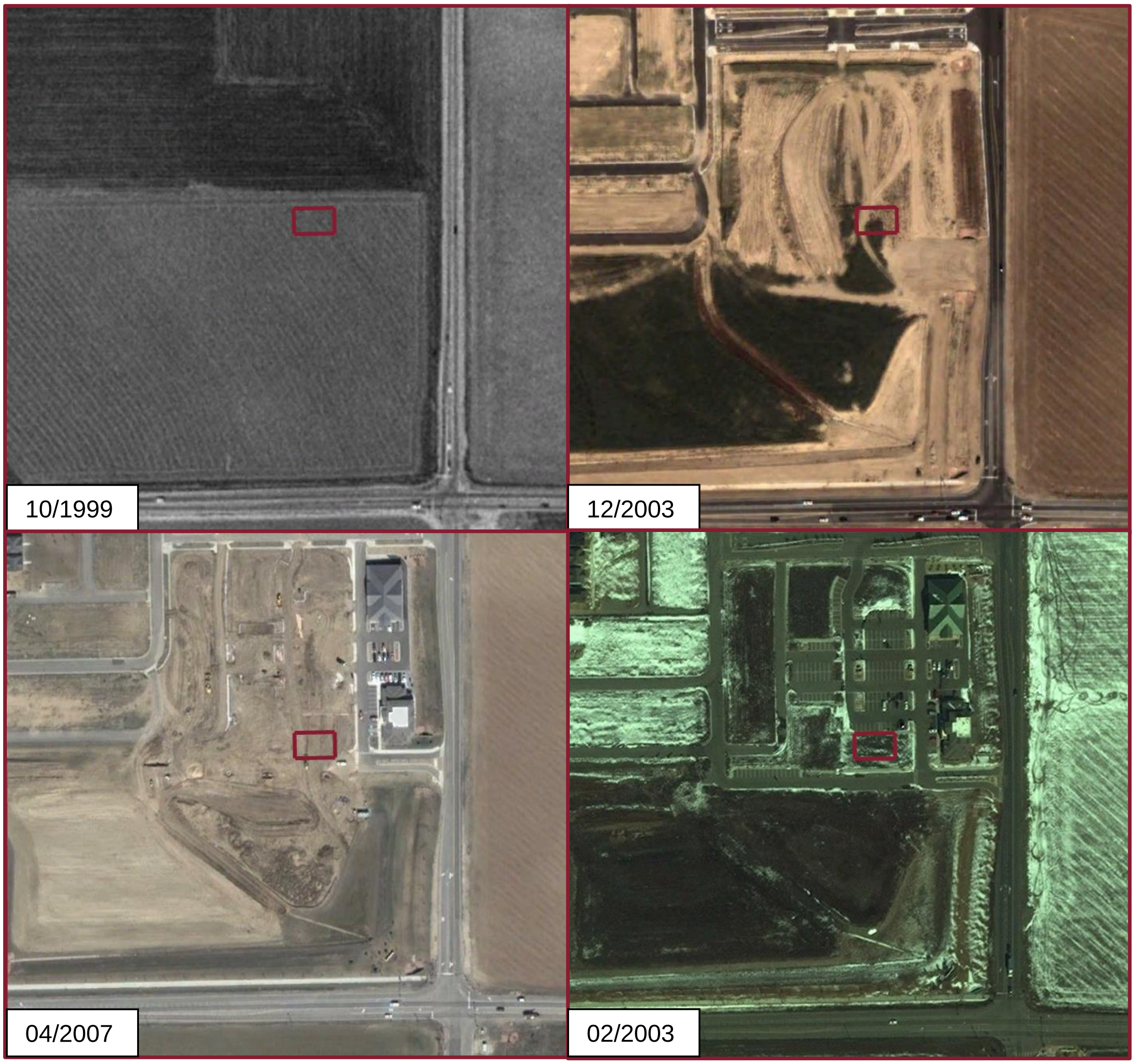
[2] Falcon Surveying, Inc. (2024) *Alta/NSPS Land Title Survey—A parcel of land situated in the southeast ¼ of section 15, township 4 north, range 69 west of the 6th P.M., Town of Berthoud, County of Larimer, State of Colorado.*

## SITE CONDITIONS

At the time of our subsurface exploration, the site generally consisted of a previously graded, empty lot vegetated with short weeds and wildflowers. Sidewalks were present on the north, east, and south sides of the site, and, based on cursory observations, seemed to be in relatively fair condition for their apparent ages. Buried utilities also bordered the perimeter of the site, evidenced by the adjacent electrical boxes and fire hydrant. Based on public locate marks, an electrical line extended through the western portion of site, through the xeriscaping to the northwest, into the electrical box on the southwest side of the site.

Site grades were very gently sloped toward the drainage swale to the east, adjacent to Berthoud Parkway. In general, the site was bordered to the west by an empty lot, to the north by parking areas, and to the east and south by private drives. Further surrounding the site, commercial and residential developments were present to the north and west, Berthoud Parkway and farmland were to the east, and undeveloped land and Colorado Highway 56 were to the west.

Review of historical aerial imagery available on Google Earth indicated that the project site, which was formerly farmland, had undergone two iterations of grading since 1999. The first iteration occurred sometime between 1999 and 2003 when the interior and perimeter roads for the adjacent residential development were constructed. In 2006, the second event occurred when the adjacent commercial properties and associated infrastructure were constructed. Sometime between 2007 and 2010, the adjacent construction was finished; the site has been undeveloped since then. Selected images of the developmental stages of the site are reproduced on the next page.




## SUBSURFACE EXPLORATION

Subsurface exploration for the project was conducted in July 2024. A total of 2 test holes were drilled with a conventional, truck-mounted drilling rig advancing 4-inch diameter, solid stem augers to evaluate the subsurface conditions and retrieve samples for laboratory testing. The test holes were advanced within the approximate building footprint area to depths of about 40 feet below existing grades, corresponding to an elevation of approximately 5,048 feet. Elevations were estimated from the provided ALTA Survey.[2] A

GROUND professional directed the subsurface exploration, logged the test holes in the field, and prepared the samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch inner diameter California liner sampler. The sampler was driven into the substrata with blows from a 140-pound hammer falling 30 inches. This procedure is similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depths at which the samples were obtained and associated penetration resistance values are shown on the test hole logs.

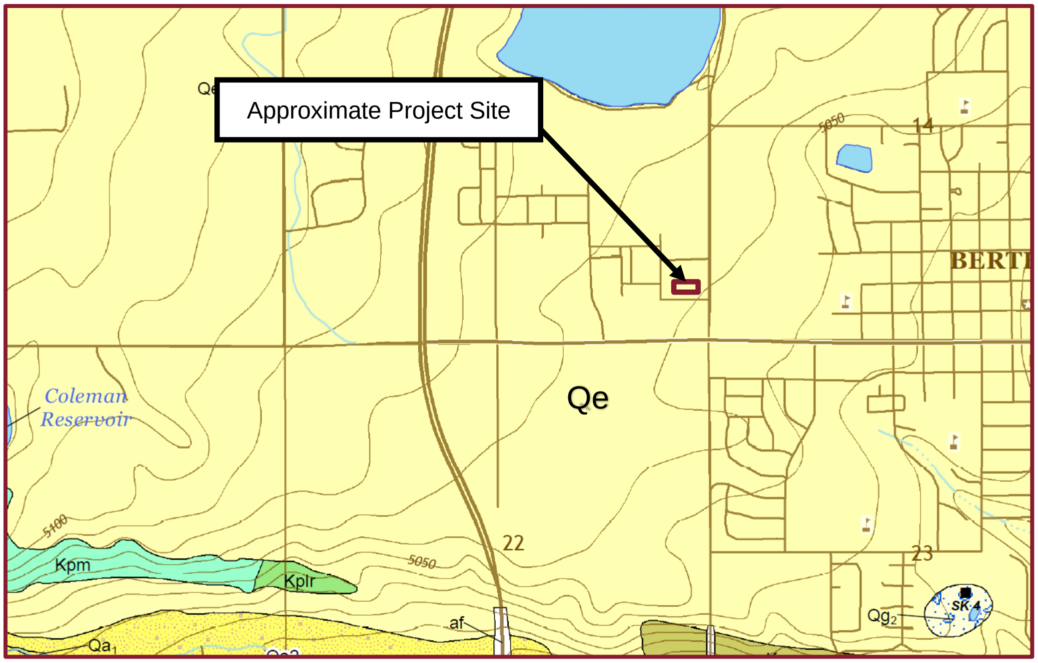
The approximate locations of the test holes are shown in Figure 1. Logs of the test holes are presented in Figure 2. A legend and notes are provided in Figure 3. Detailed logs are provided in *Appendix A*.

## LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples included standard property tests, such as natural moisture contents, dry unit weights, grain size analyses, and Atterberg limits. Swell–consolidation, water-soluble sulfate content and a suite of corrosivity tests were completed on selected samples, as well. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory testing program are summarized in Tables 1 and 2. The hydrometer plots are provided in Figures 4 and 5.

## SUBSURFACE CONDITIONS

***Geologic Setting*** Published geologic maps, e.g., Keller et. al. (2017),[3] depict the site as underlain by late Upper Pleistocene Eolian sediment (**Qe**). These surficial deposits are mapped as being underlain by Pierre Shale (**Kpm, Kpu, Kplr**). A portion of the that map is reproduced below.




In the project area, eolian (wind-blown) deposits typically consist of clays, silts, and fine sands with local medium to coarse sands; weathering typically increases the clays content of these soils. In the project area, these materials can have significant swell or collapse potentials. Collapses in these materials are generally due to hydro-consolidation of the relatively weakly cemented soil.

The Pierre Shale, in the project area, is generally described as shales, siltstones, and fine-grained sandstones, with local bentonite beds in the lower part and calcareous concretions throughout. The siltstones and shales can have significant swell potentials.

[3] Keller, S.M., Lindsey, K.O., and Morgan, M.L. (2017) *Geologic Map of the Berthoud Quadrangle—Larimer, Weld, and Boulder Counties, Colorado.* Colorado Geologic Survey. Open-File Report OF17-03, 1:24,000.

***Local Conditions*** In general the test holes penetrated about 3 feet of fill, corresponding to an elevation of about 5,085 feet, before penetrating native clays that were recognized to a depth of about 23 feet in Test Hole1 and 22 feet in Test Hole 2, corresponding to a depth of 5,065 and 5,066 feet, respectively. Below the clays, a layer of sands and gravels was encountered, and extended to a depth of about 36 feet in Test Hole 1 and 33 feet in Test Hole 2, corresponding to elevations of 5,052 and 5,055 feet, respectively. Below the sands and gravels, claystone bedrock was encountered and extended to the depths explored.

We interpret the fill materials to have been placed during overlot grading and construction of the nearby residential and commercial developments. The native clays are interpreted to be severely weathered eolian materials. The clay shale bedrock is interpreted to be materials associated with the Pierre Shale.

Fill materials were recognized in the test holes, and are likely are present across the site, given the past construction activities. (See the *Site Conditions* section of this report.) These fill soils may contain coarse gravels and cobbles, as well as similarly sized or larger pieces of construction debris even though these items where not recognized in the test holes. Delineation of the complete lateral and vertical extents of the fills at the site and their compositions was beyond our present scope of services. If more detailed information regarding fill extents and compositions at the site are of significance, they should be evaluated using test pits.

Similarly, coarse gravel and larger clasts are not well represented in small diameter liner samples collected from the test holes. Therefore, such materials may be present even where not called out in the material descriptions herein.

***Fill*** consisted of clays with fine sands. They were slightly to moderately plastic, very stiff, slightly moist, and brown in color.

***Clays*** consisted of clays with sand and sandy clays with local sands with silts and gravels. They were non- to highly plastic, very soft to stiff and loose to medium dense, slightly moist to wet, and brown to gray brown to gray in color. Secondary carbonates were noted locally.

***Sands and Gravels*** consisted of clayey sands and sands with gravels. Coarse fractions were generally fine with lesser amounts of medium to coarse sands and gravels. They were non- to moderately plastic, medium dense, very moist to wet, and brown to gray brown in color.

***Clay Shale Bedrock*** consisted of sandy clay shales with local clayey sandstones. They were non- to highly plastic, medium hard to very hard, slightly moist to very moist, and brown to brown gray in color.

***Groundwater*** was encountered at 8 feet and 7 feet below existing grades at Test Holes 1 and 2, corresponding to elevations of approximately 5,080 and 5,081 feet, respectively. More information on groundwater depths can be found in the *Excavation Considerations* section of this report. The test holes were backfilled upon drilling completion per Code of Colorado Regulations (2 CCR 402-2). Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, nearby rivers and creeks, land use, and the development of transient, perched water conditions.

The groundwater observations performed during our exploration must be interpreted carefully as they are short-term and do not constitute a groundwater study. In the event KJW Real Estate LLC desires additional/repeated groundwater level observations, GROUND should be contacted to provide a cost estimate for this additional geotechnical evaluation.

***Swell-Consolidation Testing*** of a selected sample of on-site fill soils recovered from the test holes indicated a swell of 1.2 percent under surcharge load approximating in-place overburden pressure. (See Table 1.)

## SEISMIC CLASSIFICATION

Based on extrapolation of available data to depth and our experience in the project area, we consider the area of the proposed addition likely to meet the criteria for a Seismic Site Classification of **D** according to the ASCE 7-16 (Table 1613.5.5). (Exploration and/or shear wave velocity testing to a depth of 100 feet or more was not part of our present scope of services.) If, however, a quantitative assessment of the site seismic properties is desired, then shear wave velocity testing should be performed. GROUND can provide a fee estimate for shear wave velocity testing upon request. We consider the likelihood of achieving a Site Class C to be relatively low.

Using longitude and latitude coordinates obtained from Google Earth and the ASCE 7 Hazard Tool (https://asce7hazardtool.online/), the project area is indicated to possess an $S_{DS}$ value of **0.210** and an $S_{D1}$ value of **0.091** for the site latitude and longitude and a Site Class of D.

## GEOTECHNICAL CONSIDERATIONS FOR DESIGN

The conclusions and parameters provided in this report were based on the data presented herein, our experience in the general project area with similar structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements.

Our parameters and conclusions were based on our judgment of "likely movement potentials," (i.e., the amount of movement likely to be realized if site drainage is generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the owner's willingness to accept geotechnical risk. "Maximum possible" movement estimates necessarily will be larger than those presented herein. They also have a significantly lower likelihood of being realized, in our opinion, and generally require more expensive measures to address.

We encourage KJW Real Estate, LLC, upon receipt of this report, to discuss the risks and the geotechnical information presented in this report with us. In addition to the risks and remedial approaches presented in this report, KJW Real Estate, LLC also must understand the risk-cost trade-offs addressed by the civil and structural engineering disciplines in order to direct their design team to the portion of the Higher Cost/Lower Risk–Lower Cost/Higher Risk spectrum in which this project should be designed. If KJW Real Estate, LLC does not understand these risks, it is critical that additional information or clarification be requested so that the owner's expectations reasonably can be met.

***General Geotechnical Risk*** In GROUND's opinion, there are several sources of geotechnical risk at this site. One source of geotechnical risk at this site is the relatively soft native clays. Penetration resistance values ranging from 2 blows for 12 inches to 6 blows for 12 inches were recorded in these soils and extended to depths of up to 23 feet below existing grades corresponding to an elevation of approximately 5,035 feet. Materials with such low values typically do not provide adequate bearing support for improvements like the planned building without excessive settlements.

Another source of geotechnical risk at this site is the differential consistency within the native clays. A penetration resistance value of 2 blows for 12 inches was recorded at a depth of 12 feet at Test Hole 2; at a similar depth of 14 feet at Test Hole 1, a penetration resistance value of 24 blows for 12 inches was recorded. Variations in penetration resistance values as large as these typically indicate variable support conditions; differential settlements can result where improvements are supported on such materials.

Yet another source of geotechnical risk is the presence of undocumented fill soils. Undocumented fill soils are considered to be geotechnically unsuitable to support new construction due to their unknown consistency and composition. Damaging post-construction movements have resulted where improvements have been supported directly on undocumented fill soils. Although some of the fill soils at this may have been placed in a controlled manner, records of their placement were not available for GROUND to review at the time of report preparation. Therefore, they were considered to be undocumented fill soils.

***Likely Post-Construction Movement Estimates*** Based on the data developed for this study and our experience in the project area, we estimate that post-construction movements on the order of 1½ to 2½ inches are likely, with differential movements of similar magnitudes, where improvements imposing light loads are supported directly on the existing site fill and native soils. The amount of movement will largely be dependent on the applied loads and amount of undocumented fill soil present beneath an element. Approaches that can reduce the estimates of post-construction movements are presented below.

***Building Foundation and Floor Types*** In GROUND's opinion, supporting the proposed building on drilled pier or driven pile foundation systems will provide the lowest estimates of likely post-construction foundation movement (about ½ inch, with similar differential movements over spans of about 40 feet) and will provide the least risk of excessive foundation movements. However, deep foundation systems may not be practical because they are not needed to support the structural loads and bedrock was encountered relatively deeply, about 36 feet below existing grade at Test Hole 1. Geotechnical parameters for drilled pier or driven pile foundations can be provided upon request.

Constructing the building floor as a structural floor, also supported on drilled piers or driven piles, will yield similarly low post-construction floor movement estimates. Exterior flatwork adjacent to the building, particularly at and near building entrances also should be constructed as structural floors in such cases. Geotechnical parameters for structural floors also can be provided upon request.

As a higher risk, but commonly used alternative, shallow foundations and a slab-on-grade floor could be used at this site. Shallow foundations and a slab-on-grade floor (if grades were lowered) could bear directly on the native site soils, a remedial fill section, or soils improved by installation of rammed aggregate piers. Where a remedial fill section is selected, the remedial fill section should remove and replace all the undocumented fill soils. We anticipate that the fill sections presented below, will remove and replace all of the undocumented fill soils, but greater depths of undocumented fill could be encountered, at least locally. Allowable bearing capacity for each bearing condition are tabulated below.

ALLOWABLE BEARING CAPACITIES FOR SHALLOW FOUNDATIONS

| ***Bearing Condition*** | ***Allowable Bearing Capacity*** | ***Maximum Footing Width*** | ***Estimated Post-Construction Movements*** |
|---|---|---|---|
| Existing Site Soils* | 1,000 psf | 4 feet | 1½ - 2½ inches |
| Remedial Fill Section (6 *feet of Site Soils Reworked as Fill*) | 1,000 psf | 5 feet | 1 inch |
| Remedial Fill Section (6 *feet of Select Granular Fill*) | 1,250 psf | 5 feet | 1 inch |
| Soils Improved By Rammed Aggregate Piers | > 3,000 psf** | 8 feet** | 1 inch** |

*Not in accordance with standard practice.
**Actual values will be determined by the rammed aggregate pier installer.

Due to the relatively low penetration resistance values, the native site soils will provide only a relatively low allowable bearing pressure to support the building. Where a greater bearing capacity is needed, a remedial fill section could be constructed. Such a remedial fill section should extend to a depth of at least 6 feet below existing grade and consist of select, granular fill. If a fill section of select granular fill is selected, a drain will be needed at the bottom of the fill section; underdrain parameters can be found in the *Subsurface Drainage* section of this report.

We understand, however, that constructing such a remedial fill section may not be practical. As an alternative to a remedial fill section, rammed aggregate piers could be used to improve the site soils and increase the allowable bearing capacity while reducing estimates of post-construction movements. The allowable bearing capacity, number, and depth/length of the individual elements, are determined by the specialty designer installer. The data in this report should be sufficient for the designer/installer to provide their design, but GROUND should be contacted if additional geotechnical data is needed. Rammed aggregate piers have been used successfully in the greater project area to support similar structures and appear to be the most efficient option for supporting the proposed medical office building.

For additional information, we suggest contacting a qualified and experienced designer/installer of these systems. We suggest contacting the following firms for additional information, though others may be available:

- Ground Improvement Engineering 816 – 421 – 4334
- Keller (Hayward Baker) 303 – 469 – 1136

Additional, geotechnical parameters for shallow foundation and slab-on-grade floor design are presented in the *Foundations Systems* and *Floor System* sections of this report. Likely post-construction movements for the building foundations described in the *Shallow Foundation s*ections is estimated to be on the order of 1 inch. Differential movements, likely will be on the order of ½ inch over spans of 40 feet. More detailed geotechnical parameters for design of shallow foundations and slab-on-grade floors are provided in subsequent sections of this report.

In general, we anticipate that the majority of the existing site soils will be geotechnically suitable to be reused as fill. Additional parameters and considerations regarding the suitability of the existing site soils are provided in the *Project Earthwork* section of this report.

## FOUNDATION SYSTEMS

The foundation parameters and considerations provided below were developed based on the performance expectations, geotechnical risks, and site conditions discussed in the prior sections of this report. The foundation systems used should be based on the owner's tolerance of post-construction movements and the associated cost-risk trade-offs. The use of these parameters assumes that the above discussed, system-associated risks and post-construction movement estimates are acceptable for the project.

### *Shallow Foundations*

#### *Geotechnical Parameters for Shallow Foundation Design*

1) Footings should bear on the existing site soils, a remedial fill section, or soils improved by rammed aggregate piers as discussed in the *Geotechnical Considerations for Design* section of this report.

A fill section, if selected, should extend at full thickness across the building footprint and at least **the depth of fill section** laterally beyond the footing margins, e.g., a 6-foot fill section should extend 6 feet beyond the footing margins. The fill section should be extended beneath and similarly beyond all flatwork intended to perform in the same manner as the slab-on-grade floor.

The fill section beneath the building should be laterally consistent and of uniform depth to reduce differential, post-construction foundation movements. A differential fill section will tend to increase differential movements. Considerations for fill placement and compaction are provided in the *Project Earthwork* section of this report.

The contractor should provide survey data of the excavation beneath the building indicating the depth and lateral extents of the remedial excavation.

2) The allowable bearing capacity should be selected based on the bearing condition as discussed in the *Geotechnical Considerations for Design* section of this report.

This value may be increased by ⅓ for transient loads such as wind or seismic loading. For larger footings, a lower allowable bearing pressure may be appropriate.

Immediate compression of the bearing soils as the footings are loaded to the provided allowable bearing pressure is estimated to be about ¾ inch, based on an assumption of drained foundation conditions. If foundation soils are subjected to an increase/fluctuation in moisture content, however, the effective bearing capacity will be reduced and greater post-construction movements than those estimated above may result.

This estimate of foundation movement from immediate compression of the foundation soils is a component of the total, likely, post-construction movement estimated for the buildings at this site. It is in addition to movements from post-construction volume change in the native soils underlying the site and from densification of the fill section constructed beneath the building, as discussed above.

Where rammed aggregate piers are used to improve the site soils, an allowable bearing pressure provided by the rammed aggregate pier designer/installer should be used.

To reduce differential settlements between footings or along continuous footings, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially.

3) Spread footings should have a minimum lateral dimension of **16 or more inches** for linear strip footings and **24 or more inches** for isolated pad footings. Actual footing dimensions should be determined by the structural engineer.

4) Footings should bear at an elevation **3 or more feet** below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection.

5) Continuous foundation walls should be reinforced as designed by a structural engineer to span an unsupported length of at least **10 feet**.

6) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral Loads* section of this report.

7) Connections of all types must be flexible and/or adjustable to accommodate the anticipated, post-construction movements of the structure.

8) To the extent possible, utility lines should not be routed under shallow foundations, particularly isolated pad foundations, nor in the soils supporting the foundations. Where doing so cannot be avoided, there is increased risk to both the pipe and the foundation. Measures should be included in design to protect both the footings from increased settlement (such as backfilling the utility trench with Controlled Low Strength Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material) and to protect the pipe from deformation.

   Where utility lines penetrate footings or stem walls, etc., measures should be included to accommodate the likely total and differential, post-construction movements discussed in this report. Some footings also may experience lateral displacements as structural loads are applied.

***Shallow Foundation Construction***

9) The contractor should take adequate care when making excavations not to compromise the bearing or lateral support for nearby improvements.

10) Care should be taken when excavating the foundations to avoid disturbing the supporting materials particularly in excavating the last few inches.

11) Footing excavation bottoms may expose loose, organic, or otherwise deleterious materials, including debris. Firm materials may become disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with properly compacted fill or the foundation deepened.

12) Foundation-supporting soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing bearing levels. Construction equipment should be as light as possible to limit development of this condition. The movement of vehicles over proposed foundation areas should be restricted.

13) All foundation subgrade should be compacted prior to placement of concrete.

14) Fill placed against the sides of the footings should be properly compacted in accordance with the *Project Earthwork* section of this report.

**FLOOR SYSTEMS**

The floor system parameters and considerations provided below were developed based on the performance expectations, geotechnical risks, and site conditions discussed in the prior sections of this report. The floor system used should be based on the owner's tolerance of post-construction movements and the associated cost-risk trade-offs. The use of these parameters assumes that the above discussed risks and post-construction movement estimates are acceptable for the project.

***Slab-on-Grade Floors*** The geotechnical parameters below may be used for design of slab-on-grade floors for the proposed buildings. ACI Sections 301/302/360 provide guidance regarding concrete slab-on-grade design and construction.

*Geotechnical Parameters for Design of Slab-on-Grade Floors*

1) A slab-on-grade floor system should bear on a properly compacted remedial fill section as discussed in the *Geotechnical Considerations for Design* or soils improved by rammed aggregate piers.

2) Floor slabs should be adequately reinforced. Floor slab design, including slab thickness, concrete strength, jointing, and slab reinforcement should be developed by a structural engineer.

3) An allowable vertical modulus of subgrade reaction (**Kv**) of **60 tcf** (70 pci) may be used for design of a concrete, slab-on-grade floor bearing on a properly compacted remedial fill section of site derived soils.

   If a higher modulus of subgrade reaction were needed, 3 or more feet of select, granular fill could be placed beneath the floor slab. In such a case, a (**Kv**) of **175 tcf** (202 pci) could be used.

   These values are for a 1-foot x 1-foot plate; they should be adjusted for slab dimension.

   Where rammed aggregate piers are used to improve the site soils, an allowable vertical modulus of subgrade reaction provided by the rammed aggregate pier designer/installer should be used.

4) Floor slabs should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement. Slip joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken to assure that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements.

5) Concrete slabs-on-grade should be provided with properly designed control joints.

   ACI, AASHTO, and other industry groups provide guidelines for proper design and construction concrete slabs-on-grade and associated jointing. The design and construction of such joints should account for cracking as a result of shrinkage, curling, tension, loading, and curing, as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should reflect the configuration and proposed use of the slab.

Particular attention in slab joint layout should be paid to areas where slabs consist of interior corners or curves (e.g., at column blockouts or reentrant corners) or where slabs have high length to width ratios, significant slopes, thickness transitions, high traffic loads, or other unique features. Improper placement or construction will increase the potential for slab cracking.

6) Interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. Slip joints should allow **2 inches or more** of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.

7) Post-construction heave may not displace slab-on-grade floors and utility lines in the soils beneath them to the same extent. Design of floor penetrations, connections, and fixtures should accommodate **up to 2 inches** of differential movement.

8) Moisture can be introduced into a slab subgrade during construction and additional moisture will be released from the slab concrete as it cures. A properly compacted layer of free-draining gravel, **4 or more inches** in thickness, should be placed beneath the slabs. This layer will help distribute floor slab loadings, ease construction, reduce capillary moisture rise, and aid in drainage. Selection and specification of sub-slab gravel should be coordinated with soil gas mitigation systems, where such systems are used.

The free-draining gravel should contain **less than 5 percent** material passing the No. 200 Sieve, **more than 50 percent** retained on the No. 4 Sieve, and a maximum particle size of **2 inches**.

The capillary break and the drainage space provided by the gravel layer also may reduce the potential for excessive water vapor fluxes from the slab after construction as mix water is released from the concrete.

We understand, however, that professional experience and opinion differ with regard to inclusion of a free-draining gravel layer beneath slab-on-grade floors. If these issues are understood by the owner and appropriate measures are implemented to address potential concerns including slab curling and moisture fluxes, then the gravel layer may be deleted.

9) A vapor barrier beneath a building floor slab can be beneficial with regard to reducing exterior moisture moving into the building, through the slab, but can retard downward drainage of construction moisture. Uneven moisture release can result in slab curling. Elevated vapor fluxes can be detrimental to the adhesion and performance of many floor coverings and may exceed various flooring manufacturers' usage criteria.

Per the 2006 ACI *Location Guideline*, a vapor barrier is required under concrete floors when that floor is to receive moisture-sensitive floor covering and/or adhesives, or the room above that floor has humidity control.

Therefore, in light of the several, potentially conflicting effects of the use vapor-barriers, the owner and the architect and/or contractor should weigh the performance of the slab and appropriate flooring products in light of the intended building use, etc., during the floor system design process and the selection of flooring materials. Use of a plastic vapor-barrier membrane may be appropriate for some building areas and not for others.

In the event a vapor barrier is utilized, it should consist of a minimum 15 mil thickness, extruded polyolefin plastic (no recycled content or woven materials), maintain a permeance less than 0.01 perms per ASTM E-96 or ASTM F-1249, and comply with ASTM E-1745 (Class "A"). Vapor barriers should be installed in accordance with ASTM E-1643.

Polyethylene (“poly”) sheeting (even if 15 mils in thickness which polyethylene sheeting commonly is not) does not meet the ASTM E-1745 criteria and should not be used as vapor barrier material. It can be easily torn and/or punctured, does not possess necessary tensile strength, gets brittle, tends to decompose over time, and has a relatively high permeance.

*Construction Considerations for Slab-on-Grade Floors*

10) Loose, soft, or otherwise unsuitable materials exposed on the prepared surface on which the floor slab will be cast should be excavated and replaced with properly compacted fill.

11) The fill section beneath a slab should be of uniform thickness.

12) Concrete floor slabs should be constructed and cured in accordance with applicable industry standards and slab design specifications.

13) All plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided.

## LATERAL LOADS

***Equivalent Fluid Weights and Friction Coefficients*** The following equivalent fluid pressures may be used for the design of shallow foundations, foundation walls, thrust blocks, and other shallow elements.

Note that the values provided below for the site derived fill were based on a moist unit weight ($\gamma'$) of 120 pcf and an angle of internal friction ($\phi$) of 20 degrees for the existing site fill soils and site soils reworked as properly compacted fill. The values for the CDOT Class 1 Structure Backfill were based on a moist unit weight ($\gamma'$) of 135 pcf and an angle of internal friction ($\phi$) of 34 degrees. These values are unfactored. Appropriate factors of safety should be included in design calculations.

EQUIVALENT FLUID WEIGHTS (DRAINED CONDITION)

| Backfill Material | Condition | | | Friction Coefficient |
|---|---|---|---|---|
| | Active | At-Rest | Passive | |
| Site Existing Site Soils Compacted as Fill | **59 pcf** | **80 pcf** | **200 pcf** (to a maximum of 2,000 psf) | **0.24** |
| CDOT Class 1 Structure Backfill | **39 pcf** | **60 pcf** | **435 pcf** (to a maximum of 4,350 psf) | **0.45** |

Where the full passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more.

Parameters for fill placement and compaction are provided in the *Project Earthwork* section of this report.

**WATER-SOLUBLE SULFATES**

The concentration of water-soluble sulfates measured in a selected sample of site soils was approximately 0.03 percent by weight. (See Table 2.) Such a concentration of soluble sulfates represents a **negligible** environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of "negligible," "moderate," "severe," and "very severe" as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with four classes of severity of sulfate exposure (Class 0 to Class 3) as described in the table below.

REQUIREMENTS TO PROTECT AGAINST DAMAGE TO
CONCRETE BY SULFATE ATTACK FROM EXTERNAL SOURCES OF SULFATE

| Severity of Sulfate Exposure | Water-Soluble Sulfate ($SO_4^{=}$) In Dry Soil *(%)* | Sulfate ($SO_4$) In Water *(ppm)* | Water Cementitious Ratio *(maximum)* | Cementitious Material Requirements |
|---|---|---|---|---|
| Class 0 | 0.00 to 0.10 | 0 to 150 | 0.45 | Class 0 |
| Class 1 | 0.11 to 0.20 | 151 to 1500 | 0.45 | Class 1 |
| Class 2 | 0.21 to 2.00 | 1501 to 10,000 | 0.45 | Class 2 |
| Class 3 | 2.01 or greater | 10,001 or greater | 0.40 | Class 3 |

Based on our test results and PCA and CDOT guidelines, appropriate cement conforming to one of the following requirements should be used in all concrete exposed to site soils and bedrock:

Class 0 (Negligible)

1) ASTM C150 Type I, II, III, or V.

2) ASTM C595 Type IL, IP, IP(MS), IP(HS), or IT.

## SOIL CORROSIVITY

Data were obtained to support an initial assessment of the potential for corrosion of ferrous metals in contact with earth materials at the site, based on the conditions at the time of GROUND's evaluation. The test results are summarized in Table 2.

***Reduction-Oxidation*** testing indicated a red-ox potential of approximately -120 millivolts. Such a low potential typically creates a more corrosive environment.

***Sulfide Reactivity*** testing indicated "trace" result in the local soils. The presence of sulfides in the soils suggests a more corrosive environment.

***Soil Resistivity*** In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature. Measurement of electrical resistivity indicated a value of approximately 1,200 ohm-centimeters in a sample of site soils.

***pH*** Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity.[4] Our testing indicated a pH value of about 8.9.

***Corrosivity Assessment*** The American Water Works Association (AWWA) has developed a point system scale, reproduced below, used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. At 10 points or higher, protective measures for ductile iron pipe are indicated. The soil characteristics refer to the conditions at and above pipe installation depth. We anticipate that drainage at the site after construction will be effective. Nevertheless, based on the values obtained for the soil parameters, the fill and native soils appear to comprise a severely corrosive environment for ferrous metals (20 points).

[4] American Water Works Association ANSI/AWWA C105/A21.5-05 Standard.

### Table A.1 Soil-Test Evaluation

| Soil Characteristic / Value | Points |
|---|---|
| **Redox Potential** | |
| < 0 (negative values) | 5 |
| 0 to +50 mV | 4 |
| +50 to +100 mV | 3½ |
| > +100 mV | 0 |
| **Sulfide Reactivity** | |
| Positive | 3½ |
| Trace | 2 |
| Negative | 0 |
| **Soil Resistivity** | |
| <1,500 ohm-cm | 10 |
| 1,500 to 1,800 ohm-cm | 8 |
| 1,800 to 2,100 ohm-cm | 5 |
| 2,100 to 2,500 ohm-cm | 2 |
| 2,500 to 3,000 ohm-cm | 1 |
| >3,000 ohm-cm | 0 |
| **pH** | |
| 0 to 2.0 | 5 |
| 2.0 to 4.0 | 3 |
| 4.0 to 6.5 | 0 |
| 6.5 to 7.5 | 0 * |
| 7.5 to 8.5 | 0 |
| >8.5 | 3 |
| **Moisture** | |
| Poor drainage, continuously wet | 2 |
| Fair drainage, generally moist | 1 |
| Good drainage, generally dry | 0 |

* If sulfides are present and low or negative redox-potential results (< 50 mV) are obtained, add three (3) points for this range.

If additional information or evaluation is needed regarding soil corrosivity, then the American Water Works Association or a corrosion engineer should be contacted. It should be noted, however, that changes to the site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, might alter corrosion potentials significantly.

## PROJECT EARTHWORK

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test holes. <u>Where these criteria differ from applicable municipal specifications, e.g., for trench backfill compaction along a public utility line, the latter should be considered to take precedence.</u>

***General Considerations*** Project grading should be performed as early as possible in the construction sequence to allow settlement of fills and surcharged ground to be realized to the greatest extent prior to subsequent construction.

Prior to earthwork construction, existing construction debris, vegetation, and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and properly capped.

Topsoil and other organic materials present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses. These materials should be removed and replaced where fill will be placed above them or where they will be beneath a proposed improvement.

***Use of Existing Fill and Native Soils*** Based on the samples retrieved from the test holes, we anticipate that the existing site fill and native soils that are free of organic materials, coarse cobbles, boulders, or other deleterious materials will be suitable, in general, for reuse as compacted fill.

Fragments of rock and cobbles, (as well as inert construction debris, e.g., concrete or asphalt) up to **3 inches** in maximum dimension may be included in project fills, in general. Such materials should be evaluated on a case-by-case basis, where identified during earthwork. Fragments of claystone, siltstone, and sandstone must be broken down to a soil like mass, however.

***Imported Fill Materials*** Materials imported to the site as (common) fill should be free of organic material, and other deleterious materials. Imported material should exhibit **60**

**percent or less** passing the No. 200 Sieve and a plasticity index of **10 or less**. Materials proposed for import should be approved prior to transport to the site.

*Select, Granular Fill* Material to be imported to the site as select, granular fill should meet the criteria for CDOT Class 1 Structure Backfill (tabulated below).

CDOT CLASS 1 STRUCTURE BACKFILL

| Sieve Size or Parameter | Acceptable Range |
|---|---|
| 2-inch | 100% passing |
| No. 4 | 30% to 100% passing |
| No. 50 | 10% to 60% passing |
| No. 200 | 5% to 20% passing |
| Liquid Limit | $\leq$ 35 |
| Plasticity Index | $\leq$ 6 |

Materials proposed for retaining wall backfill and/or select granular fill should be tested and approved for use a retaining wall backfill prior to import to the site.

***Fill Platform Preparation*** Prior to filling, the top **12 inches** of in-place materials on which fill soils will be placed (except for utility trench bottoms where bedding will be placed) should be scarified, moisture conditioned and properly compacted in accordance with the criteria below to provide a uniform base for fill placement.

If surfaces to receive fill expose loose, wet, soft, or otherwise deleterious material, additional material should be excavated, or other measures taken to establish a firm platform for filling. A surface to receive fill must be effectively stable prior to placement of fill, including trench bottoms prior to placement of bedding.

***General Considerations for Fill Placement*** Fill soils should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding **8 inches** in loose thickness, and properly compacted.

No fill materials should be placed, worked, rolled while they are frozen, thawing, or during poor/inclement weather conditions.

Where soils on which foundation elements will be placed are exposed to freezing temperatures or repeated freeze–thaw cycling during construction—commonly due to water ponding in foundation excavations—bearing capacity typically is reduced and/or settlements increased due to the loss of density in the supporting soils. After periods of freezing conditions, the contractor should re-work areas affected by the formation of ice to re-establish adequate bearing support.

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the specified ranges are obtained.

***Wet, Soft, or Unstable Subgrades*** Where wet, soft, or unstable subgrades are encountered, the contractor should establish a stable platform for fill placement and achieving compaction in the overlying fill soils. Therefore, excavation of the unstable soils and replacing them with relatively dry or granular material, possibly together with the use of stabilization geotextile or geogrid, may be necessary to achieve stability. Whereas the stabilization approach should be determined by the contractor, GROUND offers the alternatives below for consideration. Proof-rolling can be beneficial for identifying unstable areas.

- Replacement of the existing subgrade soils with clean, coarse, aggregate (e.g., crushed rock or “pit run” materials) or road base. Excavation and replacement to a depth of 1 to 2 feet commonly is sufficient, but greater depths may be necessary to establish a stable surface.

  On very weak subgrades, an 18- to 24-inch “pioneer” lift that is not well compacted may be beneficial to stabilize the subgrade. Where this approach is employed, however, additional settlements of up to ½ inch may result.

- Where coarse, aggregate alone does not appear sufficient to provide stable conditions, it can be beneficial to place a layer of stabilization geo-textile or geo-grid (e.g., Tencate Mirafi® RS 580*i*, or Tensar® BX 1100) at the base of the aggregate section.

  The stabilization geotextile/geogrid should be selected based on the aggregate proposed for use. It should be placed and lapped in accordance with the manufacturer's recommendations.

  Geotextile or geogrid products can be disturbed by the wheels or tracks of construction vehicles. We suggest that appropriate care be taken to maintain the effectiveness of the system. Placement of a layer of aggregate over the geotextile/geogrid prior to allowing vehicle traffic over it can be beneficial in this regard.

When a given remedial approach has been selected, the contractor should construct a test section to evaluate the effectiveness of the approach prior to use over a larger area.

***Compaction Criteria*** Soils that classify as **GP, GW, GM, GC, SP, SW, SM, or SC** in accordance with the USCS classification system (granular materials) should be compacted to **95 or more percent** of the maximum dry density at moisture contents **within 2 percent** of the optimum moisture content as determined by ASTM D1557, the modified Proctor.

Soils that classify as **ML, MH, CL, or CH** should be compacted to **at least 95 percent** of the maximum dry density at moisture contents between **1 percent below to 3 percent above** the optimum moisture content as determined by ASTM D698, the standard Proctor.

***Use of Squeegee*** Relatively uniformly graded fine gravel or coarse sand, i.e., "squeegee," or similar materials commonly are proposed for backfilling foundation excavations, utility trenches (excluding approved pipe bedding), and other areas where employing compaction equipment is difficult. In general, this procedure should not be followed for the following reasons.

Although commonly considered "self-compacting," uniformly graded granular materials require densification after placement, typically by vibration. The equipment to densify these materials is not available on many job-sites.

Even when properly densified, uniformly graded granular materials are permeable and allow water to reach and collect in the lower portions of the excavations backfilled with those materials. This leads to wetting of the underlying soils and resultant potential loss of bearing support as well as increased local heave or settlement.

Wherever possible, excavations should be backfilled with approved, on-site soils placed as properly compacted fill. Where achieving adequate compaction is difficult, then Controlled Low Strength Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material should be used for backfilling.

Where "squeegee" or similar materials are proposed for use by the Contractor, the design team should be notified by means of a Request for Information (RFI), so that the proposed use can be considered on a case-by-case basis. Where "squeegee" meets the project requirements for pipe bedding material, however, it is acceptable for that use.

***Settlements*** Settlements will occur in newly filled ground, typically on the order of 1 to 2 percent of the fill depth. This is separate from settlement of the existing soils left in place. For a 6-foot fill, for example, that corresponds to a total settlement of about 1 inch. If fill placement is performed properly and is tightly controlled, in GROUND's experience the majority (on the order of 60 to 80 percent) of that settlement typically will take place during earthwork construction, provided the contractor achieves the compaction levels indicated herein. The remaining potential settlements likely will take several months or longer to be realized, and may be exacerbated if these fills are subjected to changes in moisture content.

***Cut and Filled Slopes*** Permanent, unretained, graded slopes supported by local soils up to **5 feet** in height should be constructed no steeper than **3:1** (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well reestablished. Surface drainage should be designed to direct water away from slope faces into designed drainage pathways or structures.

Steeper slope angles and heights may be possible but will require slope-specific stability analyses based on final proposed grading plans. A geotechnical engineer should be retained to evaluate this on a case-by-case basis.

## EXCAVATION CONSIDERATIONS

***Excavation Difficulty*** Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck-mounted, geotechnical drilling equipment. Therefore, in general, we anticipate no unusual excavation difficulties in these materials for the proposed construction with conventional, heavy duty, excavating equipment. However, given the inherent nature of undocumented fill soils, materials that may be awkward or otherwise difficult to handle (e.g., relatively large pieces of construction debris) may be encountered in the undocumented fill soils.

In our experience, in the project vicinity, beds and lenses of well cemented sandstones may be present locally within the bedrock. These beds and lenses can be very hard and resistant to excavation or, for example, to advance drilled pier holes through. We understand, however, that project excavations are not anticipated to be advanced into site bedrock at this time.

***Temporary Excavations and Personnel Safety*** Excavations in which personnel will be working must comply with all applicable OSHA Standards and Regulations, particularly CFR 29 Part 1926, OSHA Standards-Excavations, adopted March 5, 1990. The contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. GROUND has provided the information in this report solely as a service to KJW Real Estate, LLC, and is not assuming responsibility for construction site safety or the contractor's activities.

The contractor should take care when making excavations not to compromise the bearing or lateral support for any adjacent, existing improvements.

Should site constraints prohibit the use of sloped excavations, temporary shoring should be used. GROUND is available to provide shoring design upon request. Stockpiling of materials should not be permitted closer than **5 feet** to the tops of temporary slopes, or **a distance equal to the depth of the excavation**, whichever is greater.

***Groundwater*** During our exploration, groundwater was encountered at Test Hole 1 at a depth of 8 feet and at Test Hole 2 at a depth of 7 feet, corresponding to elevations of 5,080 and 5,081 feet, respectively. Based on the conditions at the time of this subsurface exploration, anticipated fill section excavations at the site may encounter groundwater.

Should seepage or flowing groundwater be encountered in project excavations, the slopes should be flattened as necessary to maintain stability or a geotechnical engineer should be retained to evaluate the conditions. The risk of slope instability will be significantly increased in areas of seepage along excavation slopes.

It is possible that groundwater may be encountered in project excavations at depths both shallower and deeper than those indicated above. The contractor should be prepared to dewater the excavation during construction. Pumps adequate to discharge water and/or well points to draw down the water level may be appropriate methods. Other methods may also be necessary. The dewatering approach should ultimately be determined by the contractor based on their means and methods experience. Dewatering operations may be necessary as both temporary and long-term/permanent installations. Dewatering design should consider the potential effect on existing structures in vicinity.

If seepage or groundwater is encountered during excavation or at any time during construction, the geotechnical engineer and project team should be contacted to evaluate the conditions. The presence of groundwater in these types of situations and associated potential design changes can have an impact to both the financial and schedule components of a project.

***Surface Water*** The contractor should take proactive measures to control surface waters during construction and maintain good surface drainage conditions to direct waters away from excavations and into appropriate drainage structures. A properly designed drainage swale should be provided at the tops of the excavation slopes. In no case should water be allowed to pond near project excavations.

Temporary slopes should also be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

## UTILITY LATERAL INSTALLATION

The measures and criteria below are based on GROUND's evaluation of the local, geotechnical conditions. Where the parameters herein differ from applicable municipal requirements, the latter should be considered to govern.

***Pipe Support*** The bearing capacity of the site soils appeared adequate, in general, for support of typical utility lines. The pipes and contents are less dense than the soils which will be displaced for installation. Therefore, in general GROUND anticipates no significant pipe settlements in these materials where properly bedded from loading alone.

Trench bottoms may expose existing fill soils, or soft, loose, or otherwise deleterious materials. Firm materials may be disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with properly compacted fill.

Areas allowed to pond water will require excavation and replacement with properly compacted fill. The contractor should take particular care to ensure adequate support near pipe joints which are less tolerant of extensional strains.

Where thrust blocks are needed, the parameters provided in the *Lateral Loads* section of this report may be used for design.

***Trench Backfilling*** Some settlement of compacted soil trench backfill materials should be anticipated, even where all the backfill is placed and compacted correctly. Typical settlements are on the order of 1 to 2 percent of fill thickness. However, the need to compact to the lowest portion of the backfill must be balanced against the need to protect the pipe from damage from the compaction process. Some thickness of backfill may need to be placed at compaction levels lower than specified (or smaller compaction equipment used together with thinner lifts) to avoid damaging the pipe. Protecting the pipe in this manner can result in somewhat greater surface settlements. Therefore, although other alternatives may be available, the following options are presented for consideration:

*Controlled Low Strength Material* Because of these limitations, the entire depth of the trench (both bedding and common backfill zones) should be backfilled with "controlled low strength material" (CLSM), i.e., a lean, sand-cement slurry, "flowable fill," or similar material along all trench alignment reaches with low tolerances for surface settlements.

CLSM used as pipe bedding and trench backfill should exhibit a 28-day unconfined compressive strength between **50 to 150 psi** so that re-excavation is not unusually difficult.

Placement of the CLSM in several lifts or other measures likely will be necessary to avoid "floating" the pipe. Measures also should be taken to maintain pipe alignment during CLSM placement.

*Compacted Soil Backfilling* In areas that area tolerant of surface settlements, conventional soil backfilling may be used. Where compacted soil backfilling is employed, using the site soils or similar materials as backfill, the risk of backfill settlements entailed in the selection of this higher risk alternative must be anticipated and accepted by the facility owner.

We anticipate that the on-site soils excavated from trenches will be suitable, in general, for use as common trench backfill within the above-described limitations. Backfill soils should be free of vegetation, organic debris, and other deleterious materials. Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) coarser than 3 inches in maximum dimension should not be incorporated into trench backfills.

Soils placed for compaction as trench backfill should be conditioned to a relatively uniform moisture content, placed, and compacted in accordance with the parameters in the *Project Earthwork* section of this report.

***Pipe Bedding*** Pipe bedding materials, placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Bedding should be brought up uniformly on both sides of the pipe to reduce differential loadings.

As discussed above, the use of CLSM or similar material in lieu of granular bedding and compacted soil backfill should be considered where the tolerance for surface settlement is low. (Placement of CLSM as bedding to at least 12 inches above the pipe can protect the pipe and assist construction of a well-compacted conventional backfill, although possibly at an increased cost relative to the use of conventional bedding.)

If a granular bedding material is specified, with regard to potential migration of fines into the pipe bedding, design and installation should follow ASTM D2321, Appendix X1.8. If the granular bedding does not meet filter criteria for the enclosing soils, and we do not anticipate that it will, then non-woven filter fabric (e.g., Mirafi® 140N, or the equivalent) should be placed around the bedding to reduce migration of fines into the bedding which can result in severe, local surface settlements. Where this protection is not provided, settlements can develop/continue several months or years after completion of the project. In addition, clay or concrete cutoff walls should be installed to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along the sewer alignment which can contribute to migration of fines.

If granular bedding is specified, the contractor should not anticipate that the shallow on-site soils may be suitable for that use. Materials proposed for use as pipe bedding should be tested for suitability prior to use.

## SURFACE DRAINAGE

The site soils are relatively stable with regard to moisture content–volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movements will result primarily from the introduction of water into the soils underlying the proposed structure, hardscaping, and pavements. Based on the site surface and subsurface conditions encountered in this study, we do not anticipate a rise in the local water table sufficient to approach foundation or floor elevations. Therefore, local saturation of project foundation soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), and water flowing along constructed pathways such as bedding in utility pipe trenches. However, because of the high capillarity of the shallow site soils, elevated soil moisture in the shallow soils likely will be a permanent condition.

The following drainage measures should be followed both for during construction and as part of project design. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site drainage should be undertaken throughout the design life of the proposed facility. <u>Maintenance should be anticipated to include removal and replacement of sidewalk stones, curb and gutter, sections of pavement, etc., to restore effective drainage</u>. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

1) Wetting or drying of the underslab areas should be avoided during and after construction. Permitting increases/variations in moisture to the adjacent or supporting soils may result in increased total and/or differential movements.

2) Measures for positive surface drainage away from the building should be provided and maintained to reduce water infiltration into foundation soils. Underdrains should not be relied upon in surface drainage design to collect and discharge surface waters.

   A minimum slope of **12 inches in the first 10 feet** in the areas not covered with pavement or concrete slabs should be established. For areas covered with asphalt pavement or concrete slabs, slopes **should comply with ADA requirements where required**. Increasing slopes to a minimum of 3 percent in the first 10 feet in the areas covered with pavement or concrete slabs will reduce, but not eliminate, the potential for moisture infiltration and subsequent volume change of the underling soils.

   In no case should water be allowed to pond near or adjacent to foundation elements, hardscaping, etc.

3) Drainage also should be established <u>and maintained</u> to direct water away from sidewalks and other hardscaping as well as utility trench alignments which are not tolerant of increased post-construction movements.

   The ground surface near foundation elements should be able to convey water away readily. Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used to cover the ground surface near the foundations.

   Where the ground surface does not convey water away readily, additional post-construction movements and distress should be anticipated.

4) In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements. By the time final grading has been completed, significant volumes of water can

already have entered the subgrade, leading to subsequent distress and failures. The contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

5) In no case should water be permitted to pond adjacent to or on sidewalks, hardscaping, or other improvements as well as utility trench alignments, which are likely to be adversely affected by moisture-volume changes in the underlying soils or flow of infiltrating water.

6) Roof downspouts and drains, if used, should discharge well beyond the perimeter of the structure foundation, or be provided with positive conveyance off-site for collected waters. Downspouts should not be routed to discharge into an underdrain system.

   If roof downspouts and drains are not used, then surface drainage design should anticipate concentrated volumes of water adjacent to the buildings.

7) Irrigation water, both that applied to landscaped areas and over-spray, commonly is a significant cause of distress to improvements. Where (near-) saturated soil conditions are sustained, distress to nearby improvements should be anticipated.

   To reduce to potential for such distress, vegetation requiring watering should be located **10 or more feet** from the building perimeter, flatwork, or other improvements. Irrigation sprinkler heads should be deployed so that applied water is not introduced near or into foundation/subgrade soils. Landscape irrigation should be limited to the minimum quantities necessary to sustain healthy plant growth.

   Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation also can be beneficial for reducing the amounts of water introduced to building foundation soils, but only if the total volumes of applied water are controlled with regard to limiting that introduction. Controlling rates of moisture increase beneath the foundations, floors and other improvements should take higher priority than minimizing landscape plant losses.

Where plantings are desired within 10 feet of the building, plants should be placed in water-tight planters, constructed either in-ground or above-grade, to reduce moisture infiltration in the surrounding subgrade soils. Planters should be provided with positive drainage and landscape underdrains.

As an alternative involving only a limited increase in risk, the use of water-tight planters may be replaced by local, shallow underdrains beneath the planter beds.

8) Plastic membranes should not be used to cover the ground surface near the building without careful consideration of other components of project drainage. Plastic membranes can be beneficial to directing surface waters away from the building and toward drainage structures. However, they effectively preclude evaporation and transpiration of shallow soil moisture. Therefore, soil moisture tends to increase beneath a continuous membrane.

Where plastic membranes are used, additional shallow, subsurface drains should be installed. Perforated "weed barrier" membranes that allow ready evaporation from the underlying soils may be used.

## SUBSURFACE DRAINAGE

As a component of project civil design, properly functioning, subsurface drain systems ("underdrains") can be beneficial for collecting and discharging saturated subsurface waters. Although the subsurface drainage system anticipated for this project may consist of perimeter underdrains along the building perimeter and underdrains constructed beneath floor system, they are addressed as underdrains herein.

Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce heave or settlement of the subsurface soils, and may result in distress. Underdrains can, however, provide an added level of protection against relatively severe post-construction movements by draining saturated conditions near individual structures should they arise, and limiting the volume of wetted soil.

It is GROUND's opinion that it will be beneficial to include a perimeter underdrain system to help limit wetting of the foundation bearing soils. However, we understand that the owner and project team may consider that the reduction of risk provided by a properly constructed and maintained underdrain system does not justify the costs associated with including an underdrain. In such a case, an underdrain system can be excluded. If an underdrain system is excluded, then there will be an increased risk of the likely post-construction movements estimated in this report being exceeded. GROUND considers this increase in risk to be low, but it is not zero. Where an underdrain system is excluded, additional care should be taken to establish and maintain effective surface drainage, identify, and repair wet utility leaks in a timely manner, seal open cracks joints, and restore effective surface drainage as necessary to limit the volume of water infiltrating the site.

Where a below-grade level is added, an underdrain system should be included. If a below-grade level will underlie only a portion of the building footprint, then the underdrain system could be local to that area. Damp-proofing should be applied to the exteriors of below-grade elements. The provision of Tencate MiraFi® G-Series backing (or comparable wall drain provisions) on the exteriors of (some) below-grade elements may be appropriate, depending on the intended use.

GROUND will be available to discuss the above options and as well as other underdrain alternatives upon request.

***Geotechnical Parameters for Underdrain Design*** Where underdrains are included as a part of facility drainage design, underdrain design should incorporate the parameters below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer. Typical, cross-section details of underdrains that may be implemented for this project are provided in Figures 6 and 7.

An underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly.

1) An underdrain system for a building should consist of perforated, rigid, PVC collection pipe at least **4 inches** in diameter, non-perforated, rigid, PVC discharge pipe at least **4 inches** in diameter, free-draining gravel, and filter fabric.

2) The free-draining gravel should be naturally occurring (not recycled) material with **5 percent or less** passing the No. 200 Sieve and **50 percent or more** retained on the No. 4 Sieve, and have a maximum particle size of **2 inches**.

3) Each collection pipe should be surrounded on the sides and top (only) with **6 or more inches** of free-draining gravel.

   The gravel surrounding the collection pipe(s) should be wrapped with filter fabric (Mirafi 140N® or the equivalent) to reduce the migration of fines into the drain system.

4) The underdrain system should be designed to discharge at least **20 gallons per minute** of collected water.

5) The high point(s) for the collection pipe flow lines should be below the grade beam or shallow foundation bearing elevation as shown on the detail. Multiple high points can be beneficial to reducing the depths to which the system would be installed.

   The collection and discharge pipe for the underdrain system should be laid on a slope as determined by the underdrain designer.

   Underdrain "clean-outs" should be provided at intervals of no more than **150 feet** to facilitate maintenance of the underdrains. Clean-outs also should be provided as near as practical to collection and discharge pipe elbows of **60 degrees or more**.

6) If a below grade level is included, the underdrain system should include both a perimeter drain and lateral drains. Lateral drains should be spaced such that no point of the basement floor is more than **50 feet** horizontally from a perimeter or lateral drain collection pipe.

7) The underdrain discharge pipes should be connected to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. We suggest that collected waters be discharged directly into the storm sewer system, if possible.

8) Regular maintenance of the underdrain systems should be performed to ensure that the system continues work properly.

## EXTERIOR FLATWORK

We anticipate that the exterior of the proposed building and other portions of the site will be provided with concrete flatwork. Like other site improvements, flatwork will experience post-construction movements as soil moisture contents increase after construction and distress likely will result. The following measures will help to reduce damages to these improvements, but will not prevent all movements. Critical flatwork, which may include flatwork at entrances and exits, should be constructed as a slab-on-grade floor in a similar manner to the building's floors. Such areas should be identified by the owner.

1) Based on the plasticity of the soils and CDOT guidelines, the flatwork should be constructed on a section of properly moisture-conditioned and compacted to a depth of **at least 24 inches or a depth that removes and replaces all existing fill soils, whichever is greater.** This section assumes that KJW Real Estate, LLC will be tolerant of significant total and differential flatwork post-construction movements (on the order of several inches) and the associated maintenance costs that that are necessary to reestablish effective drainage, replace distressed flatwork, etc.

   We understand, however, that it may not be practical remove and replace all the undocumented fill soils or soft, yielding, or otherwise deleterious soils. Therefore, if the owner opts to reduce the fill section beneath the flatwork, additional post-construction movements, and additional maintenance should be anticipated. We suggest remedial earthwork should be performed to no less than 12 inches in such a case. Similarly, where existing utility lines or other site constraints limit the depth to which remedial earthwork can be accomplished, additional maintenance should be anticipated.

   In general, increasing the depth of fill beneath the flatwork will decrease the risk of post-construction movements. If performance like the well house floor is desired, then project flatwork should be constructed in a similar manner as the well house floor.

Subgrade preparation of the selected depth should extend the full width of the flatwork.

Geotechnical criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report. The contractor should be prepared to either dry the subgrade materials or moisten them, as needed, prior to compaction.

2) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The contractor should take care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. Passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the specified ranges of moisture content and relative compaction in the *Project Earthwork* section of this report. Subgrade stabilization may be cost-effective in this regard.

3) Flatwork should be provided with control joints extending to an effective depth and spaced no more than **10 feet** apart, both ways. Narrow flatwork, such as sidewalks, likely will require more closely spaced joints.

4) In no case should exterior flatwork extend to under any portion of the building where there is less than **2 inches** of vertical clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

***Concrete Scaling*** Climatic conditions in the project area including relatively low humidity, large temperature changes and repeated freeze–thaw cycles, make it likely that project sidewalks and other exterior concrete will experience surficial scaling or spalling. The likelihood of concrete scaling can be increased by poor workmanship during construction, such as “over-finishing” the surfaces. In addition, the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction, will increase the likelihood of scaling. Even use of de-icing salts on nearby roadways, from where vehicle traffic can transfer them to newly placed concrete, can be sufficient to induce scaling. Typical quality control/quality assurance tests that are performed during construction for concrete strength, air content, etc., do not provide information with regard to the properties and conditions that give rise to scaling.

We understand that some municipalities require removal and replacement of concrete that exhibits scaling, even if the material was within specification and placed correctly. The contractor should be aware of the local requirements and be prepared to take measures to reduce the potential for scaling and/or replace concrete that scales.

In GROUND's experience, the measures below can be beneficial for reducing the likelihood of concrete scaling. Which measures, if any, used should be based on cost and the owner's tolerance for risk and maintenance. It must be understood, however, that because of the other factors involved, including weather conditions and workmanship, surface damage to concrete can develop, even where all of these measures were followed. Also, the mix design criteria should be coordinated with other project requirements including criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report.

1) Maintaining a maximum water/cement ratio of 0.45 by weight for exterior concrete mixes.

2) Include Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.

3) Specify a minimum, 28-day, compressive strength of 4,500 psi for all exterior concrete.

4) Including "fibermesh" in the concrete mix also may be beneficial for reducing surficial scaling.

5) Cure the concrete effectively at uniform temperature and humidity. This commonly will require fogging, blanketing, and/or tenting, depending on the weather conditions. As long as 3 to 4 weeks of curing may be required, and possibly more.

6) Avoid placement of concrete during cold weather so that it is not exposed to freeze-thaw cycling before it is fully cured.

7) Avoid the use of de-icing salts on given reaches of flatwork through the first winter after construction.

We understand that sometimes it is not practical to implement some of these measures for reducing scaling due to safety considerations, project scheduling, etc. In such cases, where these measures are not implemented, additional costs for flatwork maintenance or reconstruction should be incorporated into project budgets.

***Frost and Ice Considerations*** Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze–thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork, and other hardscaping ("ice jacking") in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

Where hardscape movements, due to frost heave, are a design concern, e.g., at doorways, replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel should be considered or supporting the element on foundations similar to the building and spanning over a void. Detailed guidance in this regard can be provided upon request. It should be noted that where such open graded granular soils are placed, water can infiltrate and accumulate in the subsurface relatively easily, which can lead to increased settlement or heave from factors unrelated to ice formation. Therefore, where a section of open graded granular soils is placed, a local underdrain system should be provided to discharge collected water. GROUND will be available to discuss these concerns upon request.

## CLOSURE

***Geotechnical Review*** The author of this report or a GROUND principal should be retained to review project plans and specifications to evaluate whether they comply with the intent of the measures discussed in this report. The review should be requested in writing.

The geotechnical conclusions and parameters presented in this report are contingent upon observation and testing of project earthworks by representatives of GROUND. If another geotechnical consultant is selected to provide materials testing, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the parameters in this report, or by providing alternative parameters.

***Materials Testing*** KJW Real Estate, LLC, or the facility owner should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

***Limitations*** This report has been prepared for KJW Real Estate, LLC, as it pertains to design and construction of the proposed dentist office building and related improvements as described herein. It may not contain sufficient information for other parties or other purposes.

In addition, GROUND has assumed that project construction will commence by spring 2025. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be reevaluated and, as necessary, modified. **If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed herein, including changes to improvement locations, dimensions, orientations, loading conditions,**

**elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to reevaluate the conclusions and parameters presented herein.**

The geotechnical conclusions in this report relied upon subsurface exploration at a limited number of exploration points, as shown in Figure 1, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction may differ from those encountered during site exploration.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be retained at once, so that reevaluation of the conclusions for this site may be made in a timely manner. In addition, a contractor who obtains information from this report for development of his scope of work or cost estimates may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor is responsible for obtaining the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes current depths to groundwater, etc.

*ALL DEVELOPMENT CONTAINS INHERENT RISKS.* It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by KJW Real Estate, LLC Utilizing these criteria and measures herein for planning, design, and/or construction constitutes understanding and acceptance of the conclusions with regard to risk and other information provided herein, associated improvement performance, as well as the limitations inherent within such estimates.

Ensuring correct interpretation of the contents of this report by others is not the responsibility of GROUND. If any information referred to herein is not well understood, then KJW Real Estate, LLC, or other members of the design team, or the facility owner, should contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Reuse of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

GROUND appreciates the opportunity to complete this portion of the project and welcomes the opportunity to provide KJW Real Estate, LLC, with a proposal for construction observation and materials testing.

Sincerely,

**GROUND Engineering Consultants, Inc.**

Ben Fellbaum, P.G., E.I.

COLORADO LICENSED
BRIAN H. RECK
48265
PROFESSIONAL ENGINEER

8/29/2024

Reviewed by Brian H. Reck, P.G., C.E.G., P.E.


GOOGLE EARTH AERIAL IMAGE (05/22/2023)

2
Indicates test hole numbers and approximate locations.

NOT TO SCALE

GROUND ENGINEERING

JOB NO.: 24-3033

FIGURE: 1

LOCATION OF TEST HOLES

GROUND ENGINEERING

# LOGS OF THE TEST HOLES

**PROJECT:** Dental Office Building - Lot 4

**JOB NO:** 24-3033

**CLIENT:** KJW Real Estate LLC

**SITE LOCATION:** Berthoud, CO


FIGURE: 2

GROUND ENGINEERING

# LEGEND AND NOTES

**PROJECT:** Dental Office Building - Lot 4 **JOB NO:** 24-3033

**CLIENT:** KJW Real Estate LLC **SITE LOCATION:** Berthoud, CO

## *MATERIAL SYMBOLS*

FILL

CLAYS

SANDS and GRAVELS

CLAY SHALE BEDROCK

NOTE: See Detailed Logs for Material descriptions.

## *SAMPLER SYMBOLS*

**Modified California Liner Sampler**
23 / 12 Drive sample blow count indicates 23 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.

## *NOTES*

1. Test holes were drilled on 7/25/2024 with 4" solid stem auger.

2. Locations of the test holes were determined in the field using a hand held GPS device by GROUND.

3. Elevations of test holes were estimated from client provided documents and the logs of test holes are hung to elevation.

4. The test hole locations and elevations should be considered accurate only to the degree implied by the method used.

5. The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.

6. Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

7. The material descriptions on these logs are for general classification purposes only. See full text of this report for descriptions of the site materials & related information.

8. All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

## *ABBREVIATIONS*

Water Level at Time of Drilling, or as Shown

Water Level at End of Drilling, or as Shown

Water Level After 24 Hours, or as Shown

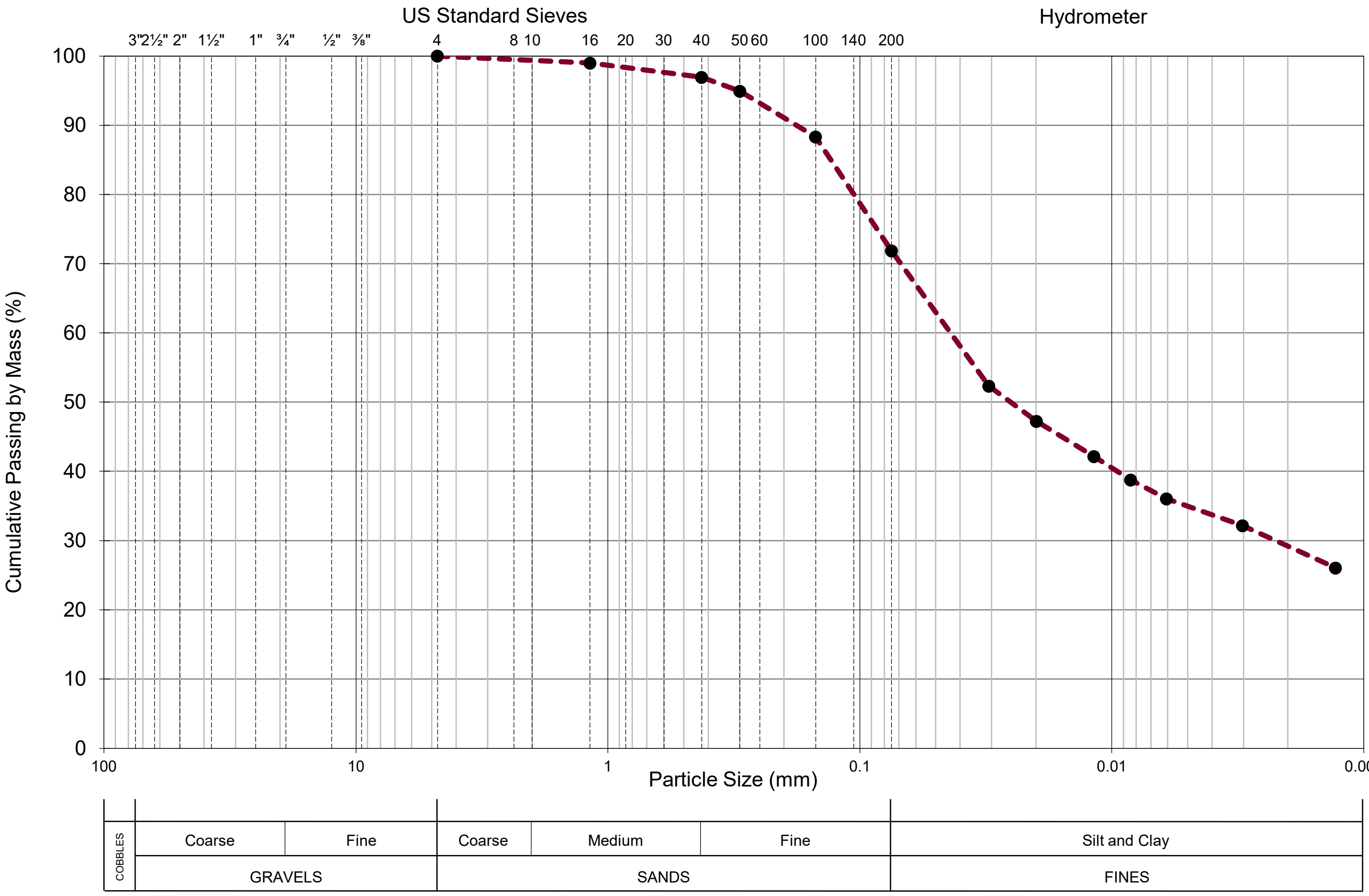
| | |
|---|---|
| NV | No Value |
| NP | Non-Plastic |

FIGURE: 3

Job No.: 24-3033

## Dental Office Building—Lot 4

## Gradation and Hydrometer (ASTM D422-63[2007])




| Coarse Gradation | | | Fine Gradation | | | Hydrometer | | Grading | |
|---|---|---|---|---|---|---|---|---|---|
| US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | Particle Size (mm) | Passing by Mass (%) | Coefficient | Value |
| 6 in | 150 | - | No. 4 | 4.75 | 100 | 0.031 | 52 | $D_{90}$ | 0.179 |
| 5 in | 125 | - | No. 8 | 2.36 | - | 0.020 | 47 | $D_{85}$ | 0.130 |
| 4 in | 100 | - | No. 10 | 2.00 | - | 0.012 | 42 | $D_{80}$ | 0.106 |
| 3 in | 75 | - | No. 16 | 1.18 | 99 | 0.008 | 39 | $D_{60}$ | 0.044 |
| 2.5 in | 63 | - | No. 20 | 0.85 | - | 0.006 | 36 | $D_{50}$ | 0.025 |
| 2 in | 50 | - | No. 30 | 0.60 | - | 0.003 | 32 | $D_{40}$ | 0.010 |
| 1.5 in | 37.5 | - | No. 40 | 0.425 | 97 | 0.001 | 26 | $D_{30}$ | 0.002 |
| 1 in | 25.0 | - | No. 50 | 0.300 | 95 | - | - | $D_{15}$ | - |
| 3/4 in | 19.0 | - | No. 60 | 0.250 | - | - | - | $D_{10}$ | - |
| 1/2 in | 12.5 | - | No. 100 | 0.150 | 88 | - | - | $D_{05}$ | - |
| 3/8 in | 9.5 | - | No. 140 | 0.106 | - | - | - | $C_u$ | - |
| No. 4 | 4.75 | 100 | No. 200 | 0.075 | 71.9 | - | - | $C_c$ | - |

*Location:* TH1 at 4 Feet
*Description:* FILL: Clay with Sand

*Classification:* (CL)s / A-6 (6)
*Liquid Limit:* 32
*Plasticity Index:* 11
*Activity:* 0.4

*Gravel (%):* 0
*Sand (%):* 28
*Silt/Clay (%):* 71.9
*< .002 mm (%):* 29

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

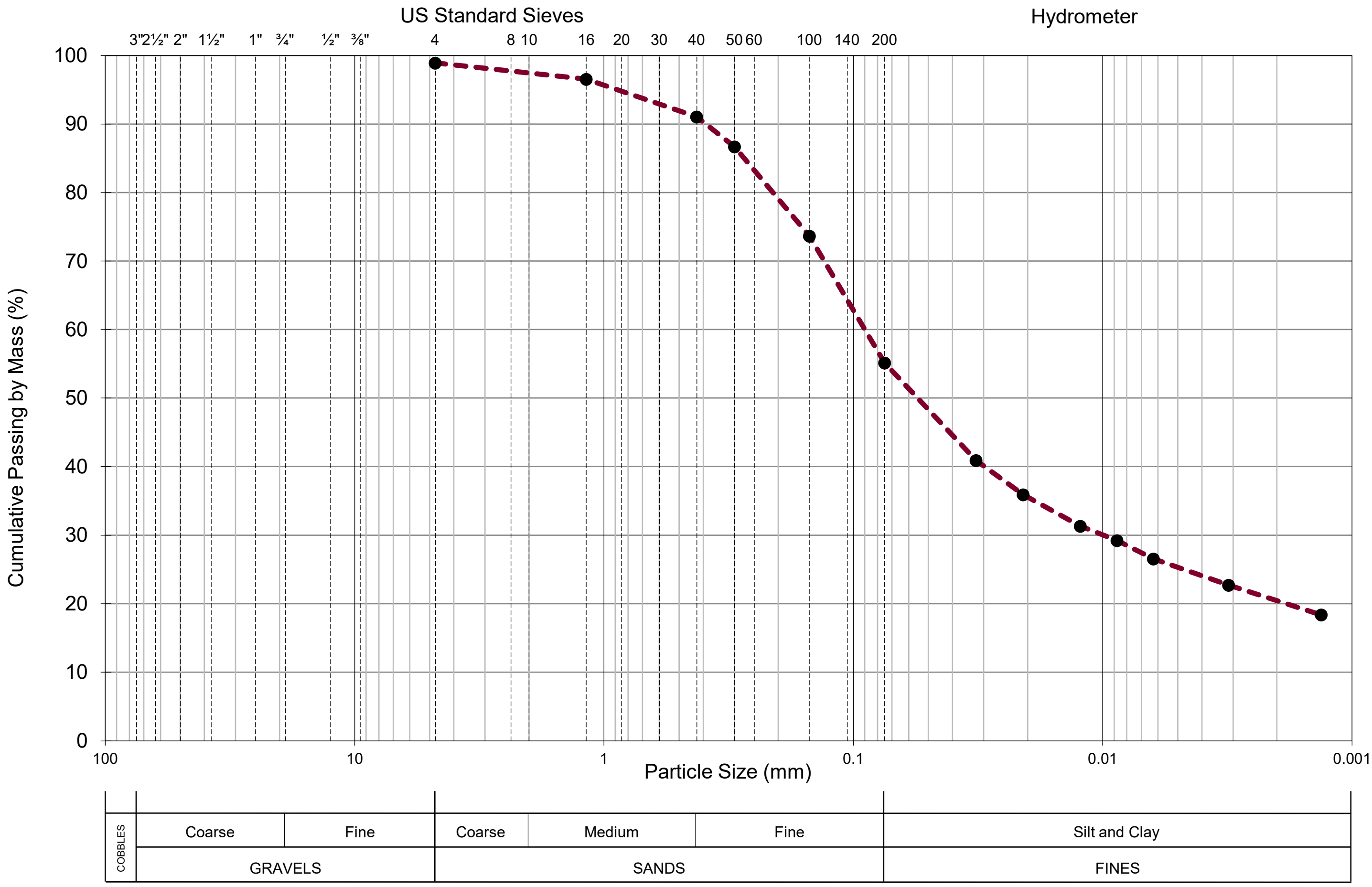
Figure 4

GROUND ENGINEERING

*Job No.:* 24-3033

## Dental Office Building—Lot 4

## Gradation and Hydrometer (ASTM D422-63[2007])




| Coarse Gradation | | | Fine Gradation | | | Hydrometer | | Grading | |
|---|---|---|---|---|---|---|---|---|---|
| US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | US Standard Sieve | Particle Size (mm) | Passing by Mass (%) | Particle Size (mm) | Passing by Mass (%) | Coefficient | Value |
| 6 in | 150 | - | No. 4 | 4.75 | 99 | 0.032 | 41 | $D_{90}$ | 0.391 |
| 5 in | 125 | - | No. 8 | 2.36 | - | 0.021 | 36 | $D_{85}$ | 0.274 |
| 4 in | 100 | - | No. 10 | 2.00 | - | 0.012 | 31 | $D_{80}$ | 0.210 |
| 3 in | 75 | - | No. 16 | 1.18 | 97 | 0.009 | 29 | $D_{60}$ | 0.090 |
| 2.5 in | 63 | - | No. 20 | 0.85 | - | 0.006 | 27 | $D_{50}$ | 0.055 |
| 2 in | 50 | - | No. 30 | 0.60 | - | 0.003 | 23 | $D_{40}$ | 0.030 |
| 1.5 in | 37.5 | - | No. 40 | 0.425 | 91 | 0.001 | 18 | $D_{30}$ | 0.010 |
| 1 in | 25.0 | - | No. 50 | 0.300 | 87 | - | - | $D_{15}$ | - |
| 3/4 in | 19.0 | - | No. 60 | 0.250 | - | - | - | $D_{10}$ | - |
| 1/2 in | 12.5 | - | No. 100 | 0.150 | 74 | - | - | $D_{05}$ | - |
| 3/8 in | 9.5 | - | No. 140 | 0.106 | - | - | - | $C_u$ | - |
| No. 4 | 4.75 | 99 | No. 200 | 0.075 | 55.1 | - | - | $C_c$ | - |

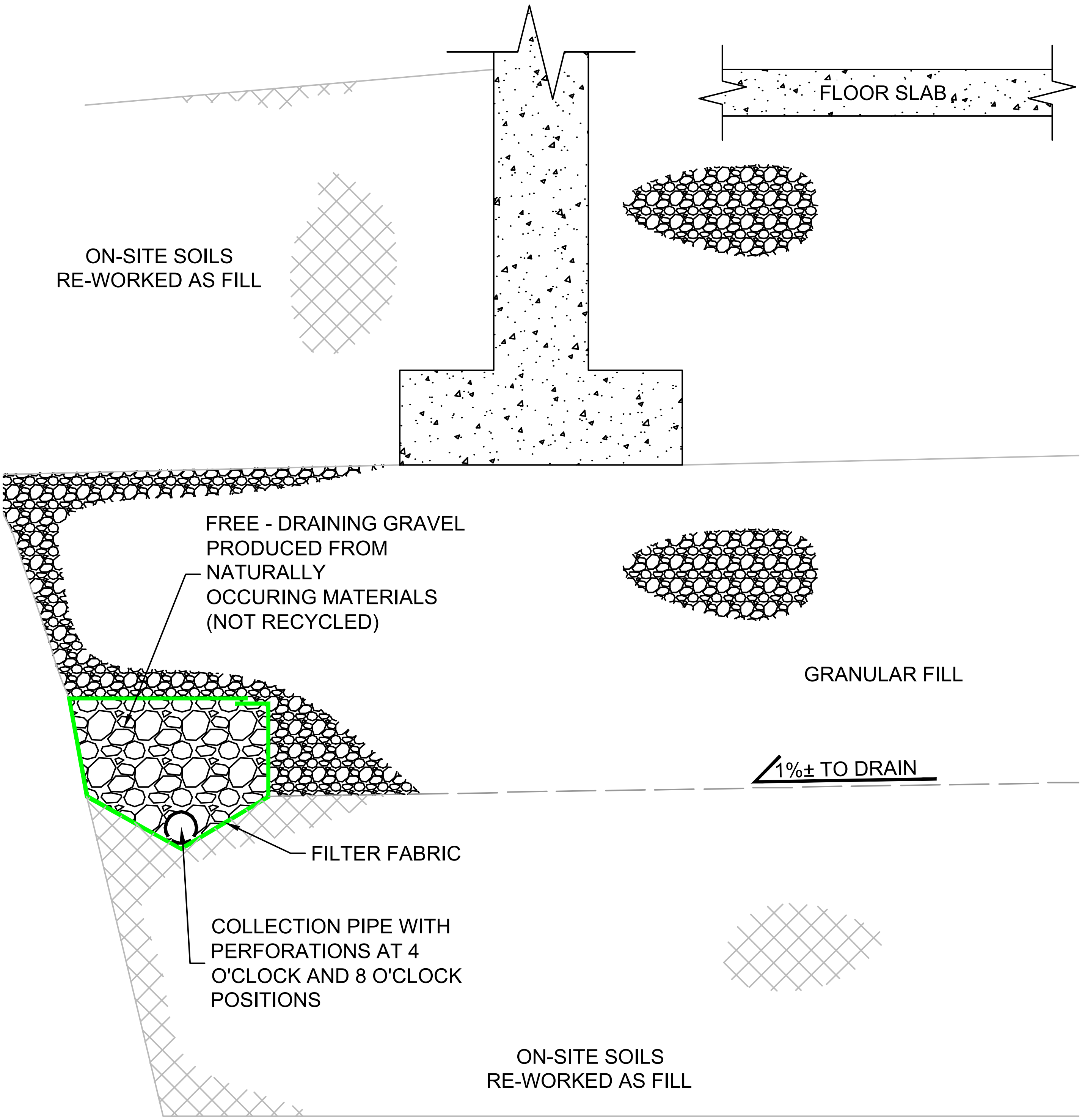
*Location:* TH2 at 12 Feet
*Description:* Sandy CLAY

*Classification:* s(CL) / A-4 (2)
*Liquid Limit:* 25
*Plasticity Index:* 9
*Activity:* 0.4

*Gravel (%):* 1
*Sand (%):* 44
*Silt/Clay (%):* 55.1
*< .002 mm (%):* 20

Results apply only to the specific items and locations referenced and at the time of testing. For the hydrometer portion of the test, a composite temperature correction and meniscus correction were applied to each reading. This report should not be reproduced, except in full, without the written permission of GROUND Engineering Consultants, Inc.

Figure 5



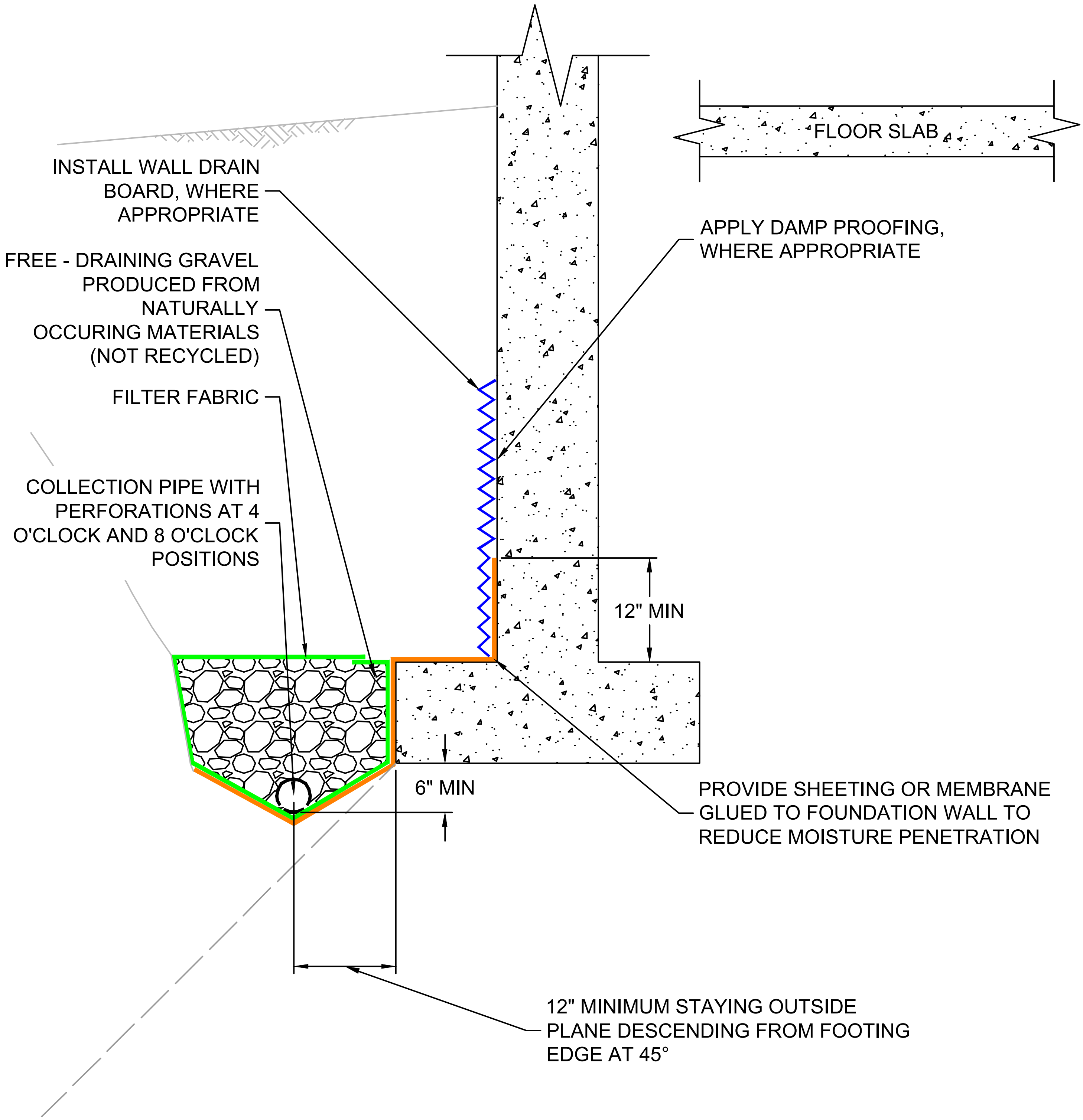

**NOTES**:

1. This is NOT a design - level drawing. it should be used solely for general information purposes only. Actual Underdrain design should be completed by others.
2. The underdrain system must be tested by the contractor after installation and backfilling to verify that it functions properly.
3. Inclusion of this figure in construction documents is done so at the document preparer's risk.
4. Reproduction of this document should be in color.

**NOT TO SCALE**
**SEE TEXT FOR ADDITIONAL INFORMATION**

| GROUND ENGINEERING | JOB NO.: 24-3033 |
|---|---|
| | FIGURE: 6 |

TYPICAL UNDERDRAIN DETAIL




**NOTES**:

1. This is NOT a design - level drawing. it should be used solely for general information purposes only. Actual Underdrain design should be completed by others.
2. The underdrain system must be tested by the contractor after installation and backfilling to verify that it functions properly.
3. Inclusion of this figure in construction documents is done so at the document preparer's risk.
4. Reproduction of this document should be in color.

**NOT TO SCALE**
**SEE TEXT FOR ADDITIONAL INFORMATION**

| GROUND ENGINEERING | JOB NO.: 24-3033 |
|---|---|
| | FIGURE: 7 |

TYPICAL UNDERDRAIN DETAIL

GROUND ENGINEERING

# Dental Office Building—Lot 4

## TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

| Sample Location | | Natural Moisture Content | Natural Dry Density | Gradation | | | Atterberg Limits | | Swell/Consolidation | | USCS Equivalent Classification | AASHTO Equivalent Classification (Group Index) | Sample Description |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Test Hole No. | Depth *(feet)* | *(%)* | *(pcf)* | Gravel *(%)* | Sand *(%)* | Fines *(%)* | Liquid Limit | Plasticity Index | Volume Change *(%)* | Surcharge Pressure *(psf)* | | | |
| 1 | 4 | 17.2 | 96.9 | 0 | 28 | 71.9 | 32 | 11 | - | - | (CL)s | A-6 (6) | CLAY with Sand |
| 1 | 9 | 11.5 | SD | 16 | 73 | 11.3 | NV | NP | - | - | (SP-SM)g | A-1-b (0) | SAND with Silt and Gravel |
| 1 | 19 | 23.1 | 101.4 | - | - | 82.1 | 43 | 22 | - | - | (CL)s | A-7-6 (18) | CLAY with Sand |
| 2 | 2 | 12.8 | 99.9 | - | - | 75.9 | 36 | 13 | 1.2 | 250 | (CL)s | A-6 (9) | FILL: Clay with Sand |
| 2 | 7 | 27.9 | 94.7 | - | - | 60.9 | 26 | 8 | - | - | s(CL) | A-4 (2) | Sandy CLAY |
| 2 | 12 | 21.6 | 108.3 | 1 | 44 | 55.1 | 25 | 9 | - | - | s(CL) | A-4 (2) | Sandy CLAY |

*SD = Sample disturbed, NV = No value, NP = Non-plastic*

*Job No. 24-3033*

### TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

| Sample Location | | Water Soluble Sulfates | pH | Redox Potential | Sulfide Reactivity | Resistivity | USCS Equivalent Classification | AASHTO Equivalent Classification (Group Index) | Sample Description |
|---|---|---|---|---|---|---|---|---|---|
| Test Hole No. | Depth *(feet)* | *(%)* | | *(mv)* | | *(ohm-cm)* | | | |
| 1 | 4 | 0.03 | - | - | - | - | (CL)s | A-6 (6) | CLAY with Sand |
| 2 | 7 | - | 8.9 | -120 | Trace | 1,200 | s(CL) | A-4 (2) | Sandy CLAY |

*Job No. 24-3033*

# Appendix A

*Detailed Logs of the Test Holes*

GROUND ENGINEERING

# TEST HOLE 1



**PROJECT:** Dental Office Building - Lot 4 **JOB NO:** 24-3033

**CLIENT:** KJW Real Estate LLC **SITE LOCATION:** Berthoud, CO

| Elevation (ft) | Depth (ft) | Material Descriptions and Drilling Notes | Sample Type | Blow Count | Natural Moisture Content (%) | Natural Dry Density (pcf) | Gradation: Gravel % | Gradation: Sand % | Gradation: Fines % | Atterberg Limits: Liquid Limit | Atterberg Limits: Plasticity Index | Swell/Consolidation (%) at Surcharge Pressure (psf) | USCS Equivalent Classification |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5058 | 0 | **FILL:** Clays with fine sands. They were slightly plastic, very stiff, slightly moist, and brown in color. | | | | | | | | | | | |
| | | **CLAYS:** Clays with sand and sandy clays with local sands with silts and gravels. They were slightly- to highly plastic, very soft to stiff and loose to medium dense, slightly moist to wet, and brown to gray brown to gray in color. Secondary carbonates were noted locally. | | | | | | | | | | | |
| 5053 | 5 | | | 6/12 | 17.2 | 96.9 | | 28 | 72 | 32 | 11 | | (CL)s |
| | | Groundwater encountered at 8 feet at the time of drilling. Groundwater encountered at 8 feet approximately 1 hours after drilling. | | | | | | | | | | | |
| 5048 | 10 | | | 6/12 | 11.5 | SD | 16 | 73 | 11 | NV | NP | | (SP-SM)g |
| 5043 | 15 | | | 24/12 | | | | | | | | | |
| 5038 | 20 | | | 10/12 | 23.1 | 101.4 | | | 82 | 43 | 22 | | (CL)s |
| | | **SANDS and GRAVELS:** Clayey sands and sands with gravels. Coarse fractions were generally fine with lesser amounts of medium to coarse sands and gravels. They were non- to moderately plastic, medium dense, very moist to wet, and brown to gray brown in color. | | | | | | | | | | | |
| 5033 | 25 | | | 18/12 | | | | | | | | | |
| 5028 | 30 | | | 12/12 | | | | | | | | | |
| 5023 | 35 | | | | | | | | | | | | |
| | | **CLAY SHALE BEDROCK:** sandy clay shales with local clayey sandstones. They were non- to highly plastic, medium hard to very hard, slightly moist to very moist, and brown to brown gray in color. | | | | | | | | | | | |
| | | | | 50/7 | | | | | | | | | |

Bottom of test hole at approx. 39.58 feet.

# GROUND ENGINEERING

# TEST HOLE 2



**PROJECT:** Dental Office Building - Lot 4 **JOB NO:** 24-3033

**CLIENT:** KJW Real Estate LLC **SITE LOCATION:** Berthoud, CO

| Elevation (ft) | Depth (ft) | Material Descriptions and Drilling Notes |
|---|---|---|
| 5058 | 0 | **FILL:** Clays with fine sands. They were slightly plastic, very stiff, slightly moist, and brown in color. |
| 5053 | 5 | **CLAYS:** Clays with sand and sandy clays with local sands with silts and gravels. They were slightly- to highly plastic, very soft to stiff and loose to medium dense, slightly moist to wet, and brown to gray brown to gray in color. Secondary carbonates were noted locally. Groundwater encountered at 7 feet at the time of drilling. |
| 5048 | 10 | |
| 5043 | 15 | |
| 5038 | 20 | |
| 5033 | 25 | **SANDS and GRAVELS:** Clayey sands and sands with gravels. Coarse fractions were generally fine with lesser amounts of medium to coarse sands and gravels. They were non- to moderately plastic, medium dense, very moist to wet, and brown to gray brown in color. |
| 5028 | 30 | |
| 5023 | 35 | **CLAY SHALE BEDROCK:** sandy clay shales with local clayey sandstones. They were non- to highly plastic, medium hard to very hard, slightly moist to very moist, and brown to brown gray in color. |
| 5018 | 40 | |

| Blow Count | Natural Moisture Content (%) | Natural Dry Density (pcf) | Gradation: Gravel % | Gradation: Sand % | Gradation: Fines % | Atterberg Limits: Liquid Limit | Atterberg Limits: Plasticity Index | Swell/Consolidation (%) at Surcharge Pressure (psf) | USCS Equivalent Classification |
|---|---|---|---|---|---|---|---|---|---|
| 20/12 | 12.8 | 99.9 | | | 76 | 36 | 13 | 1.2 (250) | (CL)s |
| 2/12 | 27.9 | 94.7 | | | 61 | 26 | 8 | | s(CL) |
| 2/12 | 21.6 | 108.3 | 1 | 44 | 55 | 25 | 9 | | s(CL) |
| 6/12 | | | | | | | | | |
| 18/12 | | | | | | | | | |
| 17/12 | | | | | | | | | |
| 47/12 | | | | | | | | | |

Bottom of test hole at approx. 40 feet.